GEOTECHNICAL EVALUATION
FIRE STATION
GOLF COURSE ROAD AND MARY DRIVE
SUPERIOR, ARIZONA

PREPARED FOR:
Willdan Associates
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March 10, 2004
Project No. 600777001

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Mr. David Gue
Willdan Associates
1717 West Northern Avenue, Suite 112
Phoenix, Arizona 85021

Subject: Geotechnical Evaluation
Fire Station
Golf Course Road and Mary Drive
Superior, Arizona

Dear Mr. Gue:

In accordance with our proposal dated February 10, 2004, Ninyo & Moore has performed a geotechnical evaluation for the above-referenced site. The attached report presents our methodology, findings, conclusions, and recommendations regarding the geotechnical conditions at the project site.

We appreciate the opportunity to be of service to you during this phase of the project. If you have any questions or comments regarding this report, please call at your convenience.

Sincerely,
NINYO & MOORE

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Date Signed 2-10-04
ARIZONA, U.S.A.

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1. INTRODUCTION

In accordance with your authorization and our proposal dated February 10, 2004, we have performed a geotechnical evaluation for the proposed Fire Station located at the southwest corner of US 60 Hwy and Airport Road in Superior, Arizona. The purpose of our evaluation was to assess the subsurface conditions at the project site in order to formulate geotechnical recommen- dations for design and construction. This report presents the results of our evaluation and our geotechnical conclusions and recommendations regarding the proposed construction.

2. SCOPE OF SERVICES

The scope of our services for the project generally included:

Reviewing readily available aerial photographs and published geologic literature, including maps and reports pertaining to the project Site and vicinity.

Marking out the boring locations and notifying Arizona Blue Stake of the boring locations prior to drilling.

Drilling, logging, and sampling two small-diameter exploratory borings to depths of about 7.5 to 13.5 feet below ground surface (bgs). The boring logs are presented in Appendix A.

Performing laboratory tests on selected soil samples obtained from the borings to evaluate in-situ moisture content and dry density, sieve analysis, Atterberg limits, response-to-wetting behavior (collapse/swell), expansion index and corrosivity characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides). The results of the laboratory testing are presented on the boring logs and/or in Appendix B.

Preparing this report presenting our findings, conclusions, and geotechnical recommendations related to excavation characteristics, building foundations, concrete slabs, subgrade support for pavements, pavement design recommendations, and re-use of on-site soils for engineered fill.

Our scope of services did not include environmental consulting services such as hazardous waste sampling or analytical testing at the site. A detailed scope of services and estimated fee for such services can be provided upon request.

3. SITE DESCRIPTION

The project site is located in Section 4, Township 2 South, Range 12 East, at the southwest corner of Golf Course Road and Mary Drive in Superior, Arizona. The approximate location of the site is depicted on the Site Location Map (Figure 1). The study site consists of undeveloped desert land with relatively heavy stands of trees. The middle portion of the site has a slightly higher elevation (on the order of 2 feet) than the remaining areas. An existing sewer crosses the site trending roughly east-west, next to the south edge of the proposed building footprint. A dirt road exists to the south of the sewer. A drainage ditch also crosses the site east-west, near the northern one-third point of the site.

According to the *United States Geological Survey (USGS), 7.5-Minute Topographic Quadrangle Map, Superior, Arizona-Pinal Co., 1981,* the site lies at an average elevation of roughly 2,650 feet relative to mean sea level (MSL). Based on the information obtained from this quadrangle map, the project site slopes gently from the east to the west.

A 1992 aerial photograph from the USGS, provided by TerraServer-USA.com shows the site in its current condition, i.e. undeveloped desert with a narrow strip cleared of vegetation where the existing dirt road and sewer are located on the site. Our limited evaluation of the aerial photograph and visual reconnaissance did not indicate any large disturbed areas that might be indicative of other past developmental activities or grading.

4. PROPOSED CONSTRUCTION

The project will generally consist of the construction of a new fire station. The fire station will have two pull-through bays for fire trucks and a third bay for an ambulance. A lean-to awning is to be constructed on the south side of the building. Eight foot wide concrete apron slabs are to extend from the east and west sides of the building. Asphalt paved driveways will be constructed from Mary Drive and Golf Course Road, and an asphalt paved parking area will be constructed on the south side of the building. The existing grades near the middle of the site will be cut about 1-1/2 to 2 feet to level the raised portion of the site.

5. FIELD EXPLORATION AND LABORATORY TESTING

On February 17, 2004, Ninyo & Moore conducted a subsurface evaluation at the site in order to evaluate the existing subsurface conditions and to collect soil samples for laboratory testing. Our evaluation consisted of the drilling, logging, and sampling of two small-diameter borings. The borings were advanced using a CME-75 truck-mounted drill rig. The borings were planned to be drilled to an approximate depth of 15 feet bgs. However, auger and sampler refusal was encountered at approximate depths of 7.5 feet in boring B-1, and 13.6 feet in boring B-2. Bulk and relatively undisturbed soil samples were collected at selected intervals. Detailed descriptions of the soils encountered are presented on the boring logs in Appendix A. The general locations of the borings are shown on the Boring Location Map (Figure 2).

The soil samples collected from our drilling activities were transported to the Ninyo & Moore laboratory in Phoenix, Arizona, for geotechnical laboratory analysis. The analysis included in-situ moisture content and dry density, sieve analysis, Atterberg limits, response-to-wetting behavior (collapse/swell), expansion index and corrosivity characteristics (including pH, minimum electrical resistivity, soluble sulfates, and chlorides). The results of the in-situ moisture content and dry density testing are presented on the boring logs in Appendix A. A description of the laboratory test methods and the remaining test results are presented in Appendix B.

6. GEOLOGY AND SUBSURFACE CONDITIONS

The geology and subsurface conditions at the site are described in the following sections.

6.1. Geologic Setting

The project site is located in the Sonoran Desert Section of the Basin and Range physiographic province, which is typified by broad alluvial valleys separated by steep, discontinuous, subparallel mountain ranges. The mountain ranges generally trend north-south and northwest-southeast. The basin floors consist of alluvium with thickness extending to several thousands of feet.

The basins and surrounding mountains were formed approximately 10 to 13 million years ago during the mid- to late-Tertiary. Extensional tectonics resulted in the formation of horsts (mountains) and grabens (basins) with vertical displacement along high-angle normal faults. Intermittent volcanic activity also occurred during this time. The surrounding basins filled with alluvium from the erosion of the surrounding mountains, as well as from deposition from rivers. Coarser-grained alluvial material was deposited at the margins of the basins near the mountains. The surficial geology of the site is described as middle Pleistocene to latest Pliocene alluvium with less abundant talus and eolian deposits (Reynolds, 1988). Descriptions of the soils encountered during our evaluation are presented in the following section.

6.2. Subsurface Conditions

Our knowledge of the subsurface conditions at the project site is based on our field exploration and laboratory testing, and our understanding of the general geology of the area. The following paragraph provides a generalized description of the materials encountered. More detailed descriptions are presented on the boring logs in Appendix A.

Alluvium was encountered at the surface of the borings and extended to the total depths explored. The alluvium generally consisted of clayey sand layers underlain by clayey silt and silty sand layers. Auger and sampler refusal was encountered in these two borings at approximate depths of 7.5 and 13.6 feet. Caliche nodules and moderate to well cemented caliche layers were observed within the alluvium.

6.3. Groundwater

Groundwater was not encountered in our borings. Based on well data from the Arizona Department of Water Resources, the approximate depth to groundwater is about 20 feet bgs in the vicinity of the site. Groundwater levels can fluctuate due to seasonal variations, irrigation, groundwater withdrawal or injection, and other factors. In general, groundwater is not expected to be a constraint to the construction of the project.

7. GEOLOGIC HAZARDS

The following sections describe potential geologic hazards at the site, including land subsidence and earth fissures, faulting and seismicity, surface rupture, and liquefaction.

7.1. Land Subsidence and Earth Fissures

Groundwater depletion due to groundwater pumping has caused land subsidence and earth fissures in numerous alluvial basins in southern Arizona. It has been estimated that subsidence has affected more than 3,000 square miles and has caused damage to a variety of engineered structures and agricultural land (Schumann and Genualdi, 1986). From 1948 to 1983, excessive groundwater withdrawal has been documented in several alluvial valleys where groundwater levels have been reportedly lowered by up to 500 feet. With such large depletions of groundwater, the alluvium has undergone consolidation resulting in large areas of land subsidence.

In Arizona, earth fissures are generally associated with land subsidence and pose an ongoing geologic hazard. Earth fissures generally form near the margins of geomorphic basins where significant amounts of groundwater depletion have occurred. Reportedly, earth fissures have also formed due to tensional stress caused by differential subsidence of the unconsolidated alluvial materials over buried bedrock ridges and irregular bedrock surfaces (Schumann and Genualdi, 1986).

Based on our field reconnaissance and review of the referenced material, there are no known earth-fissures underlying the subject site. Based on our research, the closest earth fissure to the site is located approximately 29 miles to the west of the project site, where water levels have dropped by approximately 100 to 300 feet. Continued groundwater withdrawal in the area may result in subsidence and the formation of new fissures or the extension of existing fissures. While the future occurrence of land subsidence and earth fissures cannot be accurately predicted, these phenomena are not expected to be a constraint to the construction of this project.

7.2. Faulting and Seismicity

The site lies within the Sonoran Zone, which is a relatively stable tectonic region located in southwestern Arizona, southeastern California, southern Nevada, and northern Mexico (Euge et al., 1992). This zone is characterized by sparse seismicity and few Quaternary faults. Based on our field observations, review of pertinent geologic data and analysis of aerial photographs, faults are not located on or adjacent to the property. The closest fault to the site is the Sugar Loaf fault zone, located approximately 35 miles to the northwest of the site (Pearthree, 1998). Up to 5 meters of displacement has occurred along this fault within upper and uppermost Pleistocene deposits, but middle Holocene deposits are not displaced.

Based on a Probabilistic Seismic Hazard Assessment for the Western United States, issued by the USGS (1999), the site is located in a zone where the peak ground accelerations that have a 10 percent, 5 percent, and 2 percent probability of being exceeded in 50 years are 0.07g, 0.10g and 0.16g, respectively. Seismic design parameters according to the 1997 Uniform Building Code (UBC) are presented in the following table.

Table 1 – Seismic Design Parameters

Parameter	Value	1997 UBC Reference
Seismic Zone Factor, Z	0.075	Table 16 – I
Soil Profile Type	S_D	Table 16 – J
Seismic Coefficient C_a	0.12	Table 16 – Q
Seismic Coefficient Cv	0.18	Table 16 – R
Near-Source Factor, N_a	1.0	Table 16 – S
Near-Source Factor, N_v	1.0	Table 16 – T
Seismic Source Type	C	Table 16 – U

7.3. Liquefaction Potential

Based on the SPT values recorded in our borings at various depths, the lack of near-surface water, and the low ground motion hazard (relatively low peak ground accelerations), the likelihood or potential for liquefaction of site soils is considered to be negligible and is, therefore, not a design consideration.

8. CONCLUSIONS

Based on the results of our subsurface evaluation, laboratory testing, and data analysis, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations of this report are incorporated into project design and construction, as appropriate. Geotechnical considerations include the following:

- The on-site soils should generally be excavatable to foundation depths with conventional earth moving construction equipment in good working condition.
- Based on the results of the field and laboratory evaluations, it is our opinion that the proposed structure can be founded on shallow foundations proportioned for moderate bearing pressures supported on adequately moisture-conditioned and compacted engineered fill. Concrete slab-on-grade floors, pavements, and exterior concrete flatwork should also be founded on adequately moisture-conditioned and compacted engineered fill.
- Imported soils and soils generated from on-site excavation activities that exhibit a very low to low expansion potential can generally be used as engineered fill.
- Groundwater was not observed in our borings. The static groundwater table is anticipated to be about 40 feet bgs or deeper.
- No known or potential geologic hazards are reported underlying, or adjacent to, the site.
- Corrosivity test results indicate that subgrade soils at the site may be corrosive to ferrous metals and the sulfate content of the soils present a negligible sulfate exposure to concrete.

9. RECOMMENDATIONS

The following sections present our geotechnical recommendations for the proposed construction. If the proposed construction is changed from that discussed in this report, or if future modifications are proposed, Ninyo & Moore should be contacted for additional recommendations.

9.1. Earthwork

The following sections provide our earthwork recommendations for this project. In general, the earthwork specifications contained in Maricopa Association of Governments (MAG), *Uniform Standard Specifications and Details for Public Works Construction*, as modified by the Town of Superior are expected to apply, except as noted.

9.1.1. Excavations

Our evaluation of the excavation characteristics of the on-site materials is based on the results of two exploratory borings, our site observations, and our experience with similar materials. In our opinion, excavation of the on-site materials can generally be accomplished to the expected shallow foundation and utility depths with conventional earthmoving equipment in good operating condition. However, scattered caliche filaments and nodules, caliche cementation and cobbles were encountered in the borings, which could be more difficult to excavate depending on the actual size and degree of cementation encountered during construction.

The contractor should provide safely sloped excavations or an adequately constructed and braced shoring system, in compliance with Occupational Safety and Health Administration regulations, for employees working in an excavation that may expose employees to the danger of moving ground. If material is stored or equipment is operated near an excavation, stronger shoring should be used to resist the extra pressure due to superimposed loads.

9.1.2. Grading, Fill Placement, and Compaction

Vegetation and debris from the clearing operation should be removed from the site and disposed of at a legal dumpsite. Demolition debris, if any, should also be removed from the site and disposed of at a legal dumpsite. Obstructions that extend below the finish grade, if present, should be removed and the resulting holes filled with compacted soil.

The geotechnical consultant should carefully evaluate any areas of loose or soft and wet soils prior to placement of grade-raise fill or other construction. Drying or overexcava-

tion of some materials may be appropriate, in addition to the earthwork preparation recommendations indicated below.

Soils generated from on-site grading activities that exhibit relatively low plasticity indices and very low to low expansive potential are generally suitable for reuse as engineered fill. Relatively low plasticity indices are defined as a value of 20 or less. Very low to low expansive potential soils are defined as having an Expansion Index (by ASTM D 4829) of 50 or less, or a swell potential of 1.5 percent or less, when tested in accordance with ASTM D-4546-96, Method B, and remolded to 98 percent of its standard Proctor (ASTM D 698-00) maximum dry density and at a moisture content of two percent below its optimum. Laboratory tests performed during this evaluation indicated Expansion Indices of 13 and 38, demonstrating a very low to low expansion potential. The two Atterberg Limits tests indicated plasticity indices of 5 and 20.

Suitable fill should not include organic material, clay lumps, construction debris, rock particles, and other non-soil fill materials larger than 6 inches in dimension. This material should be broken up to a size of 6 inches or less, disposed of off site or placed in non-structural areas.

As stated previously, our exploratory borings disclosed near-surface alluvial deposits, consisting primarily of clayey sand, silty sand and clayey silt. The alluvium exhibited some potential for collapse upon inundation with water. In addition, some of the near-surface soils did not exhibit adequate density to safely support structural footing loads. Accordingly, we recommend that new building foundations be founded on adequately moisture-conditioned and compacted engineered fill, extending 2 or more feet below the lowest foundation bearing elevation. This new fill should be placed in horizontal lifts approximately 9 inches in loose thickness and compacted by appropriate mechanical methods, to a relative compaction of 95 percent or more in accordance with American Society of Testing and Materials (ASTM) D 698-00 at a moisture content of approximately 2 percent above the optimum moisture or higher. The overexcavation should extend laterally to 4 feet or more beyond the foundation footprint.

In addition, we recommend that new slabs-on-grade, pavements, and exterior flatwork be supported on 12 or more inches of adequately moisture-conditioned and compacted structural fill. The fill thickness should be measured from the bottom of the base material and should be compacted by appropriate mechanical methods, to a relative compaction of 95 percent or more in accordance with ASTM D 698-00 at a moisture content of approximately 2 percent above its optimum moisture or higher.

Following the overexcavation as described above, and prior to the placement of new fill, the excavation bottoms should be carefully evaluated by the geotechnical consultant. Based on this evaluation, additional remediation may be needed. This could include scarification of the exposed surface. This additional remediation, if needed, should be addressed by the geotechnical consultant during the earthwork operations.

9.1.3. Imported Fill Material

Imported fill, where utilized, should consist of clean, granular material with a very low or low expansion potential. Import material in contact with ferrous metals or concrete should preferably have low corrosion potential (minimum resistivity greater than 2,000 ohm-cm, chloride content less than 25 parts per million [ppm], and soluble sulfate content of less than 0.1 percent). The geotechnical consultant should evaluate such materials and details of their placement prior to importation.

9.2. Foundations

We recommend utilizing conventional, shallow spread or continuous footings for the proposed fire station building. The shallow footings should be supported at a depth of 18 or more inches below the lowest adjacent grade on 2 or more feet of moisture-conditioned, recompacted material, as described in Section 9.1.2. Continuous footings should have a width of 12 or more inches, and isolated spread footings should have a width of 24 or more inches. Spread or continuous footings should be reinforced in accordance with the recommendations of the structural engineer. Footings may be designed using an allowable bearing capacity of up to 2,000 pounds per square foot (psf) for static conditions.

Total and differential settlement of new footings of up to about 1 inch and 1/2 inch, respectively, may occur. Distortions of about 1 inch (vertical) over 40 feet (horizontal) are possible.

Structural footings bearing on engineered fill materials and subject to lateral loading may be designed using an ultimate coefficient of friction of 0.35 (total frictional resistance equals the coefficient of friction multiplied by the dead load). An ultimate passive resistance value of 250 psf of depth can be used. The lateral resistance can be taken as the sum of the frictional resistance and passive resistance, provided that the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces. The footings should preferably be proportioned such that the resultant force from lateral loadings falls within the kern (i.e., middle one-third).

9.3. Floor Slabs

For new slabs, the design is typically the responsibility of the structural engineer; however, from a geotechnical standpoint we recommend that the floor slab have a thickness of 4 or more inches. Floor slabs subject to heavy wheel or equipment loads should have a thickness of 6 or more inches. The slabs should be reinforced with No. 3 steel reinforcing bars placed 24 inches on-center (each way) in the middle one-third of the slab height. The placement of the reinforcement in the slab is vital for satisfactory performance. The slabs should be underlain by 4 or more inches of base course or leveling pad. We recommend that this material conform to Arizona Department of Transportation (ADOT) Class 2 specifications. In general, Class 2 material consists of 100 percent of the material passing the 1-1/2 inch sieve, 90 to 100 percent of the material passing the 1 inch sieve, 35 to 55 percent of the material passing the No. 8 sieve, and no more than 8 percent passing the No. 200 sieve. A moisture-retarding system and/or vapor barrier should also be placed beneath new floor slabs in areas where moisture sensitive floor coverings are anticipated. We recommend that the mix design for the slabs be provided to Ninyo & Moore for review. Quality control testing and evaluations should be provided during placement of the floor slabs.

The new floor slabs should either be constructed so that they "float" independent of the foundations or be designed to remain structurally connected to the foundations. Soils underlying the slabs should be moisture conditioned and compacted in accordance with the recommendations presented in Section 9.1.2. Joints should be constructed at intervals designed by the structural engineer to help reduce random cracking of the slab.

9.4. Pavements

For the paved areas, we anticipate that asphaltic concrete will be utilized, except for the aprons next to the proposed building, which are to be constructed of Portland cement concrete. The following sections present our recommendations for both pavement sections.

9.4.1. Flexible Pavement Design

Although no laboratory R-values were evaluated for this project, correlated R-values were found using the laboratory information obtained from the sieve analyses and Atterberg limits tests. A correlated R-value of 25 was found for boring B-1 from a 1 to 2-1/2 foot depth, and a correlated R-value of 59 was found for boring B-2 from a 3.5 to 4.8 foot depth. Using the lower correlated R-value of 25 for new pavements, based on Figure 202.02-2 of the ADOT Preliminary Engineering and Design Manual, and correlating the R-value and a seasonal variation factor of 2.1 taken from Table 202.02-4, a resilient modulus of 9,000 pounds per square inch was estimated. This subgrade resilient modulus (M_R) should be representative of the native clayey sand soils encountered in borings B-1 and B-2, and engineered fill materials derived from the native soils in the proposed pavement areas.

The AASHTO method was used to evaluate the asphalt pavement sections for the project. The design is based on the following input parameters:

Design Period	20 years
ESALs (standard-duty pavement):	211,000
ESALs (heavy-duty pavement):	433,000
Reliability:	85 percent
Overall Deviation:	0.45
Resilient Modulus:	9,000 psi
Structural Number	
(Standard duty pavement)	2.38
(Heavy-duty pavement)	2.67
Initial Serviceability	4.2
Terminal Serviceability:	2.25

The following table presents the layer materials and thicknesses recommended for the bituminous pavement areas of the project:

Table 2 – Recommended Asphalt Pavement Sections

Layer	Material	Thickness (Inches) Standard Duty	Thickness (Inches) Heavy Duty
Bituminous Wearing Course	MAG Section 710	1.5	1.5
Bituminous Leveling Course	MAG Section 710	1.5	2.5
Aggregate Base Course	MAG Section 702 (See Table 3 below)	6.0	6.0

The aggregate base mentioned above should meet Section 702 of the MAG specifications and/or any Town of Superior requirements, as shown in Table 3.

Table 3 – Recommended Aggregate Base Gradation

Sieve Size (per ASTM D422-63)	Percent Passing by Weight
1 1/8 inch	100
No. 4	38-65
No. 8	25-60
No. 30	10-40
No. 200	3-12
P.I. Max.	5

Aggregate base materials should be compacted to a relative compaction of 98 percent or more of the maximum dry density, as evaluated by ASTM D 698-00, at a moisture content of approximately 2 percent above the optimum or higher.

A bond coat should be utilized between the base course and wearing course when either 48 hours have elapsed between placement of the bituminous courses, or if the surface of the pavement has been contaminated with dirt, dust or foreign material. The bond coat should not be applied unless the existing pavement surface is free of moisture.

The new pavements should be designed to provide positive surface drainage. A surface slope of 1 or more percent is recommended. The pavement surface should be smooth, free of roller marks or depressions, and should not contain any irregularities which would pond or impede water flow.

Ninyo & Moore should be advised if new information relative to the design traffic becomes available in order to further evaluate the pavement sections provided.

9.4.2. Rigid Pavement Design

Portland cement concrete should have a thickness of 6 or more inches in heavy-duty pavement areas. In parking areas not subject to truck traffic (standard-duty pavement areas), the concrete pavement thickness can be reduced to 5 inches.

Concrete pavements should have longitudinal and transverse joints that meet the applicable requirements of the MAG Uniform Standard Specifications and the Town of Superior requirements. Concrete pavements should be underlain by a 4 inch thick aggregate base course.

The minimal reinforcement for the concrete pavement areas should be evaluated by the structural engineer based on specific loading conditions. For the Portland cement concrete and asphalt pavement sections discussed above, we recommend the underlying subgrade soils be prepared as described in Section 9.1.2 of this report.

9.5. Concrete Flatwork

To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer. We recommend that exterior concrete flatwork be supported on 12 or more inches of adequately moisture-conditioned and compacted engineered fill as described in Section 9.1.2 of this report. Positive drainage should be established and maintained adjacent to flatwork.

9.6. Corrosion

The corrosion potential of the on-site materials was analyzed to evaluate its potential effect on the foundations and structures. Corrosion potential was evaluated using the results of laboratory testing of a sample obtained during our subsurface evaluation that was considered representative of soils at the subject site.

Laboratory testing consisted of pH, minimum electrical resistivity, and chloride and soluble sulfate contents. The pH and minimum electrical resistivity tests were performed in general accordance with Arizona Test 236b, while sulfate and chloride tests were performed in accordance with Arizona Test 733 and 722, respectively. The results of the corrosivity tests are presented in Appendix B.

The soil pH value of the sample tested from boring B-2 was 7.7, which is considered to be alkaline. The minimum electrical resistivity measured in the laboratory was 510 ohm-cm, which may be corrosive to ferrous metals. The chloride content of the sample tested was measured to be 690 ppm, which may also be considered corrosive to ferrous metals. The soluble sulfate content of the soil sample was measured to be less than 0.01 percent, which is considered to represent a negligible sulfate exposure for concrete.

The results of the laboratory testing indicate that the on-site materials could be corrosive to ferrous metals. Therefore, special consideration should be given to the use of heavy gauge, corrosion protected, underground steel pipe or culverts, if any are planned. As an alternative, plastic pipe or reinforced concrete pipe could be considered. A corrosion specialist should be consulted for further recommendations.

9.7. Concrete

Laboratory chemical tests performed on a selected sample of on-site soil indicated a sulfate content of less than 0.01 percent by weight. Based on the following UBC table, the on-site soils should be considered to have a negligible sulfate exposure to concrete.

Table 4 – UBC Requirements for Concrete Exposed to Sulfate-Containing Soil

Sulfate Exposure	Water-Soluble Sulfate (SO_4) in Soil, Percentage by Weight	Cement Type	Maximum Water-Cementitious Materials Ratio, by Weight, Normal-Weight Aggregate Concrete[1]	Minimum f'_c, Normal-Weight and Lightweight Aggregate Concrete, psi x 0.00689 for MPa
Negligible	0.00 - 0.10	--	--	--
Moderate[2]	0.10 - 0.20	II, IP(MS), IS (MS)	0.50	4,000
Severe	0.20 - 2.00	V	0.45	4,500
Very severe	Over 2.00	V plus pozzolan[3]	0.45	4,500

[1] A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing (Table 19-A-2).
[2] Seawater.
[3] Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

Notwithstanding the sulfate test results and due to the limited number of chemical tests performed, as well as our experience with similar soil conditions and local practice, we recommend the use of Type II cement for construction of concrete structures at this site. Due to potential uncertainties as to the use of reclaimed irrigation water, or topsoil that may contain higher sulfate contents, pozzolan or admixtures designed to increase sulfate resistance may be considered.

The concrete should have a water-cementitious materials ratio no greater than 0.45 by weight for normal weight aggregate concrete. The structural engineer should ultimately select the concrete design strength based on the project specific loading conditions. Higher strength concrete may be selected for increased durability and resistance to slab curling and shrinkage cracking.

We also recommend that crack control joints be provided in slabs in accordance with the recommendations of the structural engineer to reduce the potential for distress due to minor soil movement and concrete shrinkage. We further recommend that concrete cover over reinforcing steel for slabs on grade and foundations be in accordance with UBC 1907.7.1. The structural engineer should be consulted for additional concrete specifications.

9.8. Site Drainage

Surface drainage should be provided to divert water away from the building and off of paved surfaces. Surface water should not be permitted to drain toward the structure or to pond adjacent to footings or on paved areas. Positive drainage is defined as a slope of 2 or more percent for a distance of 5 or more feet away from the structures. Roof gutters should be installed on structures. Downspouts should discharge to drainage systems away from structures, pavements, and flatwork. Soil improvements below the new grade slabs and pavement sections should be sloped toward the edges of these areas.

9.9. Construction Observation and Testing

During construction operations, we recommend that a qualified geotechnical consultant perform observation and testing services for the project. These services should be performed to evaluate exposed subgrade conditions, including the extent and depth of overexcavation, to

evaluate the suitability of proposed borrow materials for use as fill and to observe and test placement and compaction of fill soils. Qualified subcontractors utilizing appropriate techniques and construction materials should perform construction of the proposed improvements.

10. LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

11. SELECTED REFERENCES

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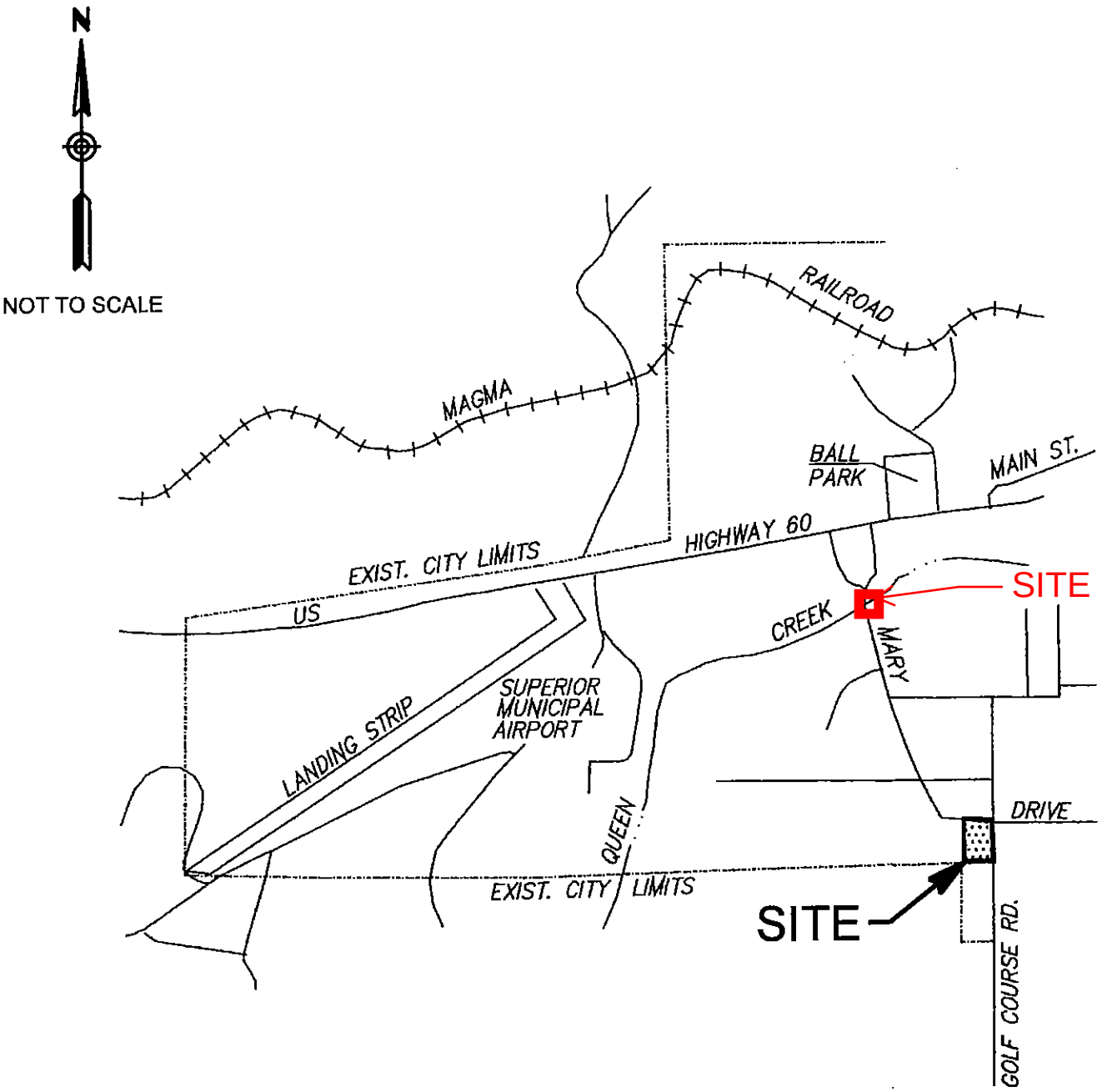
AERIAL PHOTOGRAPHS

Source	Date	Scale/Resolution
USGS (www.terraserver.com)	1997	NA



NOTE:
QUEEN CREEK IS ABOUT 0.36 MILES FROM THE STUDIED LOCATION

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SITE LOCATION MAP

SUPERIOR FIRE STATION
GOLF COURSE & MARY DRIVE
SUPERIOR, ARIZONA

PROJECT NO.	DATE
600777001	3/04

FIGURE 1

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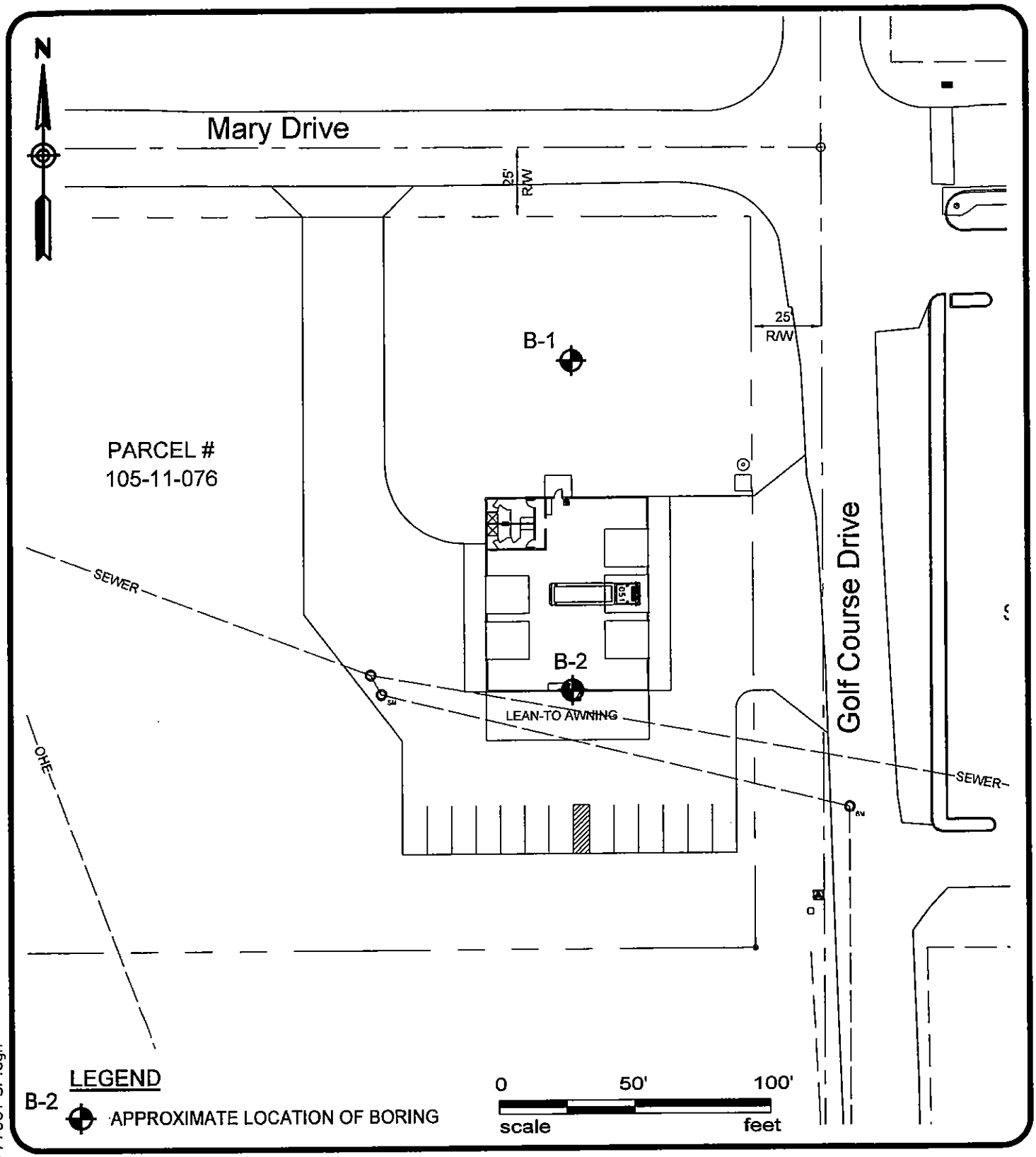

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BORING LOCATION MAP

SUPERIOR FIRE STATION
GOLF COURSE & MARY DRIVE
SUPERIOR, ARIZONA

PROJECT NO.	DATE	FIGURE
600777001	3/04	2

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test SPT) Spoon
Disturbed drive samples of earth materials were obtained by means of a SPT spoon sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The spoon was driven up to 18 inches into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with American Society for Testing and Materials (ASTM) D 1586-99. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the spoon, bagged, sealed, and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler
The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586-99. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

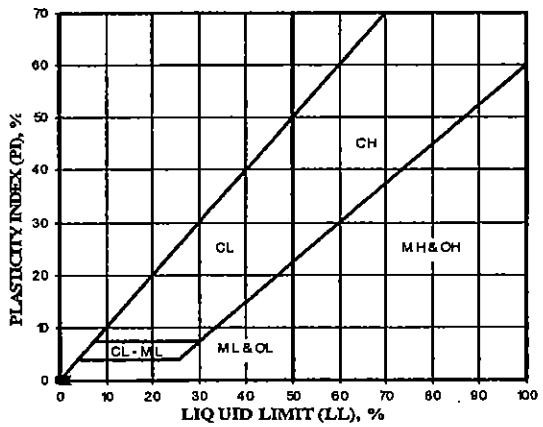
U.S.C.S. METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS		SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (More than 1/2 of soil >No. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction <No. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 1/2 of soil <No. 200 sieve size)	SILTS & CLAYS Liquid Limit <50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit >50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

GRAIN SIZE CHART

CLASSIFICATION	RANGE OF GRAIN SIZE	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL Coarse Fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND Coarse Medium Fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.074 4.76 to 2.00 2.00 to 0.420 0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

PLASTICITY CHART



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U.S.C.S. METHOD OF SOIL CLASSIFICATION

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0	■							Bulk sample.
		■						Modified split-barrel drive sampler.
		╳						No recovery with modified split-barrel drive sampler.
		▬						Sample retained by others.
		◢						Standard Penetration Test (SPT).
5		◢						No recovery with a SPT.
		▯	XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
		◢						No recovery with Shelby tube sampler.
		▯						Continuous Push Sample.
				♀				Seepage.
10				▽				Groundwater encountered during drilling.
				▼				Groundwater measured after drilling.
						▦	SM	ALLUVIUM: Solid line denotes unit change.
								Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Sheared Bedding Surface
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG		
EXPLANATION OF BORING LOG SYMBOLS		
PROJECT NO.	DATE Rev. 01/03	FIGURE A-0

DATE DRILLED 2/17/04 BORING NO. B-1

GROUND ELEVATION ±2950' MSL SHEET 1 OF 1

METHOD OF DRILLING CME-75, 6.5 Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Automatic Hammer) DROP 30"

SAMPLED BY ESZ LOGGED BY ESZ REVIEWED BY RDL

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0		39	8.1	95.4		SC	ALLUVIUM: Dark brown, damp, medium dense, clayey fine to coarse SAND; few gravel; scattered caliche filaments and nodules; high plasticity clay.
		8				ML	Brown, damp, stiff, clayey SILT; few fine sand; scattered caliche nodules.
5		79/11"	6.4	110.7		SC	Light brown, damp, very dense, clayey fine to coarse SAND; little gravel; few cobbles.
							Total Depth = 7.5 feet. (Auger refusal on boulder or cobble.) Groundwater not encountered. Backfilled on 2/17/04.
10							
15							
20							

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BORING LOG

Superior Fire Station
Superior, Arizona

PROJECT NO.	DATE	FIGURE
600777001	3/04	A-1

DATE DRILLED 2/17/04 BORING NO. B-2

GROUND ELEVATION ±2952' MSL SHEET 1 OF 1

METHOD OF DRILLING CME-75, 6.5 Hollow-Stem Auger

DRIVE WEIGHT 140 lbs. (Automatic Hammer) DROP 30"

SAMPLED BY ESZ LOGGED BY ESZ REVIEWED BY RDL

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SC	ALLUVIUM: Reddish brown, damp, dense, clayey fine to coarse SAND; few gravel; scattered caliche filaments; high plasticity clay.
		29					
		90/10"	10.1	90.5		SM	Light reddish brown, damp, very dense, silty fine to coarse SAND; scattered moderately cemented caliche nodules and layers.
5							
		50/5"					
							Brown, dense, few gravel, scattered caliche nodules.
		47					
10							Difficult drilling from 10 to 13.5 feet.
		50/1"					Strongly cemented caliche layer.
							Total Depth = 13.6 feet. (Sampler refusal on caliche layer.) Groundwater not encountered. Backfilled on 2/17/04.
15							
20							

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BORING LOG

Superior Fire Station
Superior, Arizona

PROJECT NO.	DATE	FIGURE
600777001	3/04	A-2

APPENDIX B

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with American Society for Testing and Materials (ASTM) D 2488-00. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory excavations were evaluated in general accordance with ASTM D 2937-00. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422-63. The grain-size distribution curves are shown on Figures B-1 and B-2. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318-00. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-3.

Hydroconsolidation (Settlement Potential) Tests
Hydroconsolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435-96. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures B-4 through B-6.

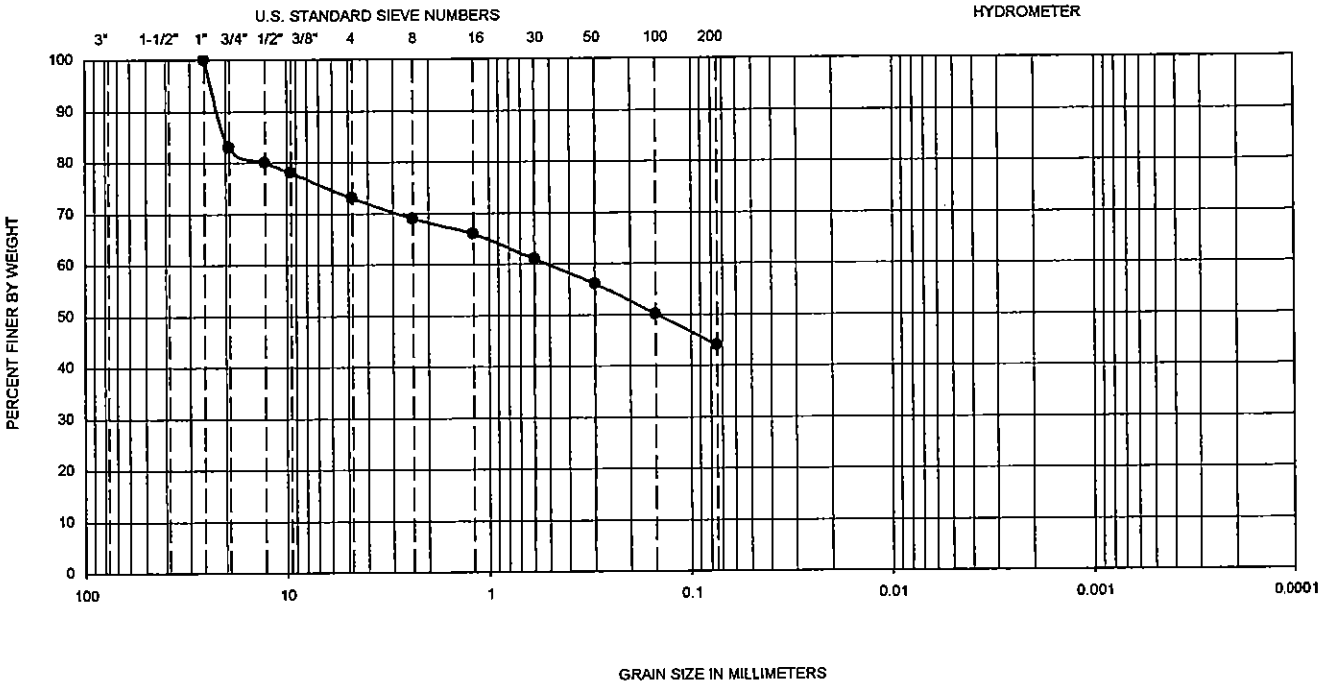
Expansion Index Tests
The expansion index of selected materials was evaluated in general accordance with U.B.C. Standard No. 18-2. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inun-

dated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on FigureB-7.

Soil Corrosivity Tests

Soil pH and minimum resistivity tests were performed on representative samples in general accordance with Arizona Test 236b. The chloride content of selected samples was evaluated in general accordance with Arizona Test 722. The sulfate content of selected samples was evaluated in general accordance with Arizona Test 733. The test results are presented on Figure B-8.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-1	1.0-2.5	--	--	--	--	--	--	--	--	44	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

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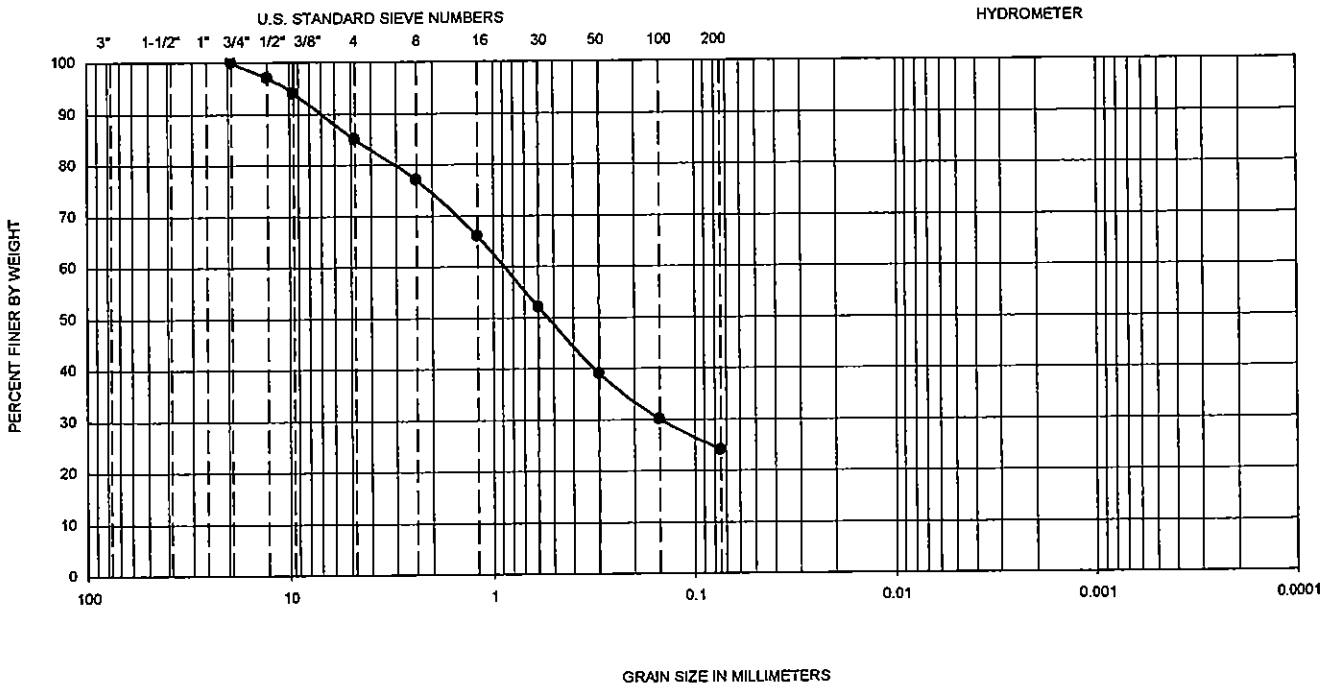
GRADATION TEST RESULTS

SUPERIOR FIRE STATION
GOLF COURSE ROAD & MARY DRIVE
SUPERIOR, ARIZONA

PROJECT NO.	DATE	FIGURE
600777001	3/04	B-1

SVB1@1-2.5

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	B-2	3.5-4.8	–	–	–	–	–	–	–	–	24	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63

Ninyo & Moore

GRADATION TEST RESULTS

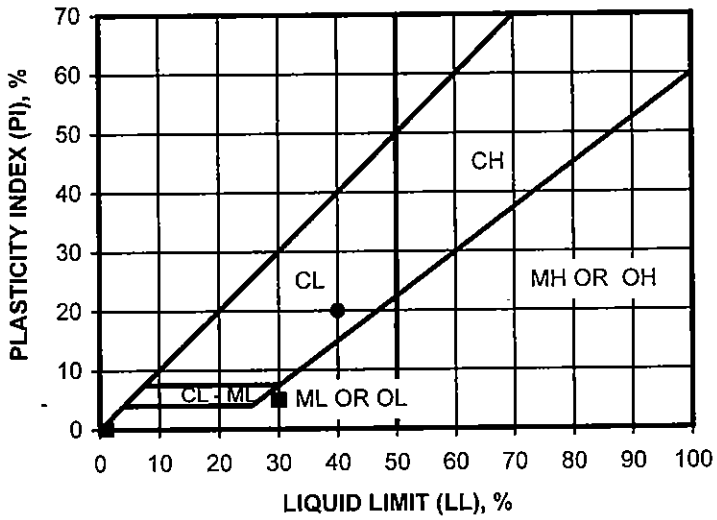
SUPERIOR FIRE STATION
GOLF COURSE ROAD & MARY DRIVE
SUPERIOR, ARIZONA

PROJECT NO.	DATE	FIGURE
600777001	3/04	B-2

SVB2@3.5-4.8

SYMBOL	LOCATION	DEPTH (FT)	LL (%)	PL (%)	PI (%)	U.S.C.S. CLASSIFICATION (Minus No. 40 Sieve Fraction)	U.S.C.S. (Entire Sample)
●	B-1	1.0-2.5	40	20	20	CL	SC
■	B-2	3.5-4.8	30	25	5	ML	SM

NP - Indicates non-plastic



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-00

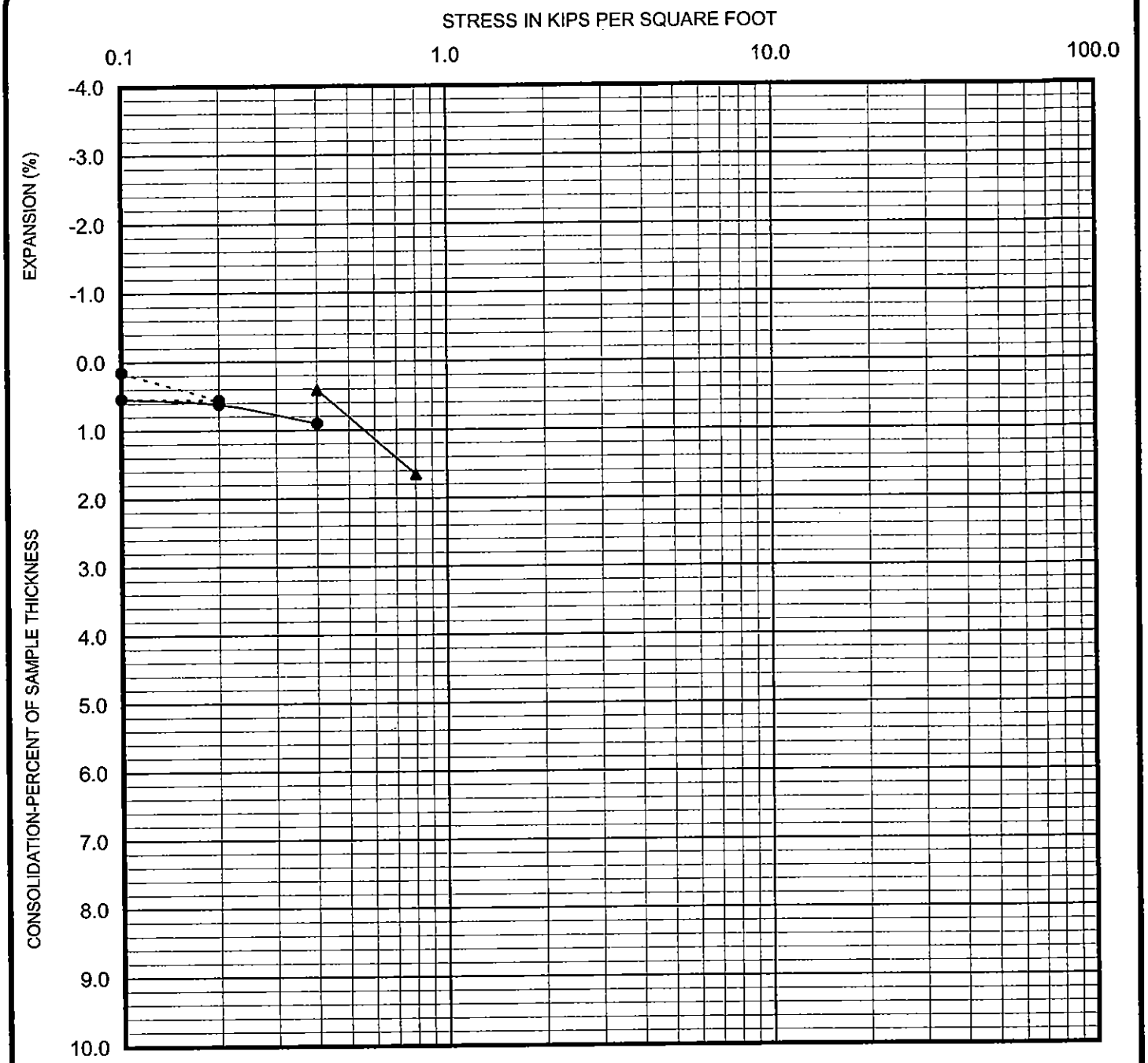
Ninyo & Moore

ATTERBERG LIMITS TEST RESULTS

SUPERIOR FIRE STATION
GOLF COURSE ROAD & MARY DRIVE
SUPERIOR , ARIZONA

PROJECT NO.	DATE	FIGURE
600777001	3/04	B-3

ATB2@3.5-4.8



Seating Cycle	Boring No. B-1
Loading Prior to Inundation	Depth (ft.) 1.0-2.5
Loading After Inundation	Soil Type SC
Rebound Cycle	PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

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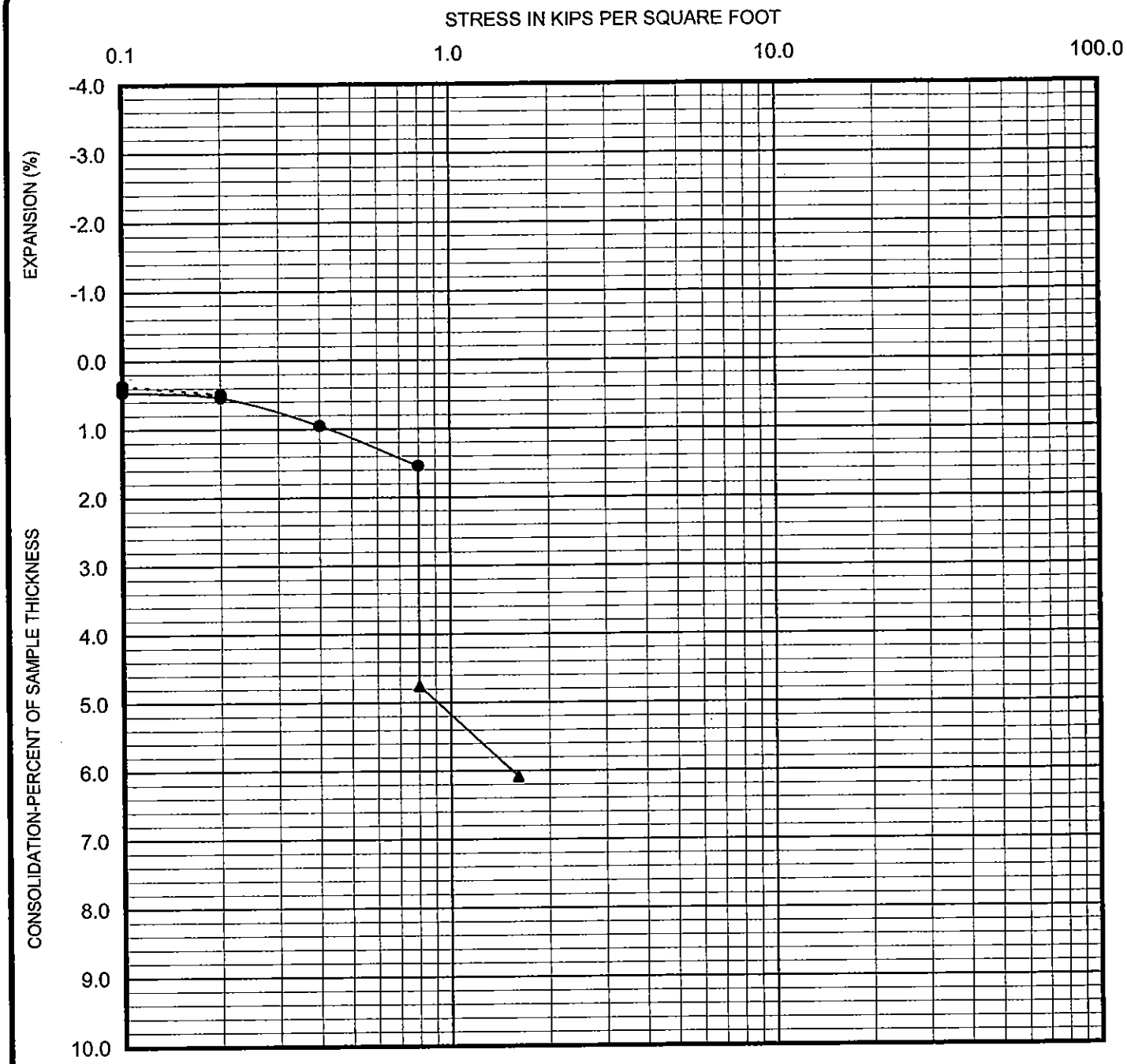
CONSOLIDATION TEST RESULTS

SUPERIOR FIRE STATION
GOLF COURSE ROAD & MARY DRIVE
SUPERIOR, ARIZONA

PROJECT NO.	DATE	FIGURE
600777001	3/04	B-4

CNB1@1-2.5



Seating Cycle	Boring No. B-1
Loading Prior to Inundation	Depth (ft.) 6.0-7.4
Loading After Inundation	Soil Type SC
Rebound Cycle	PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

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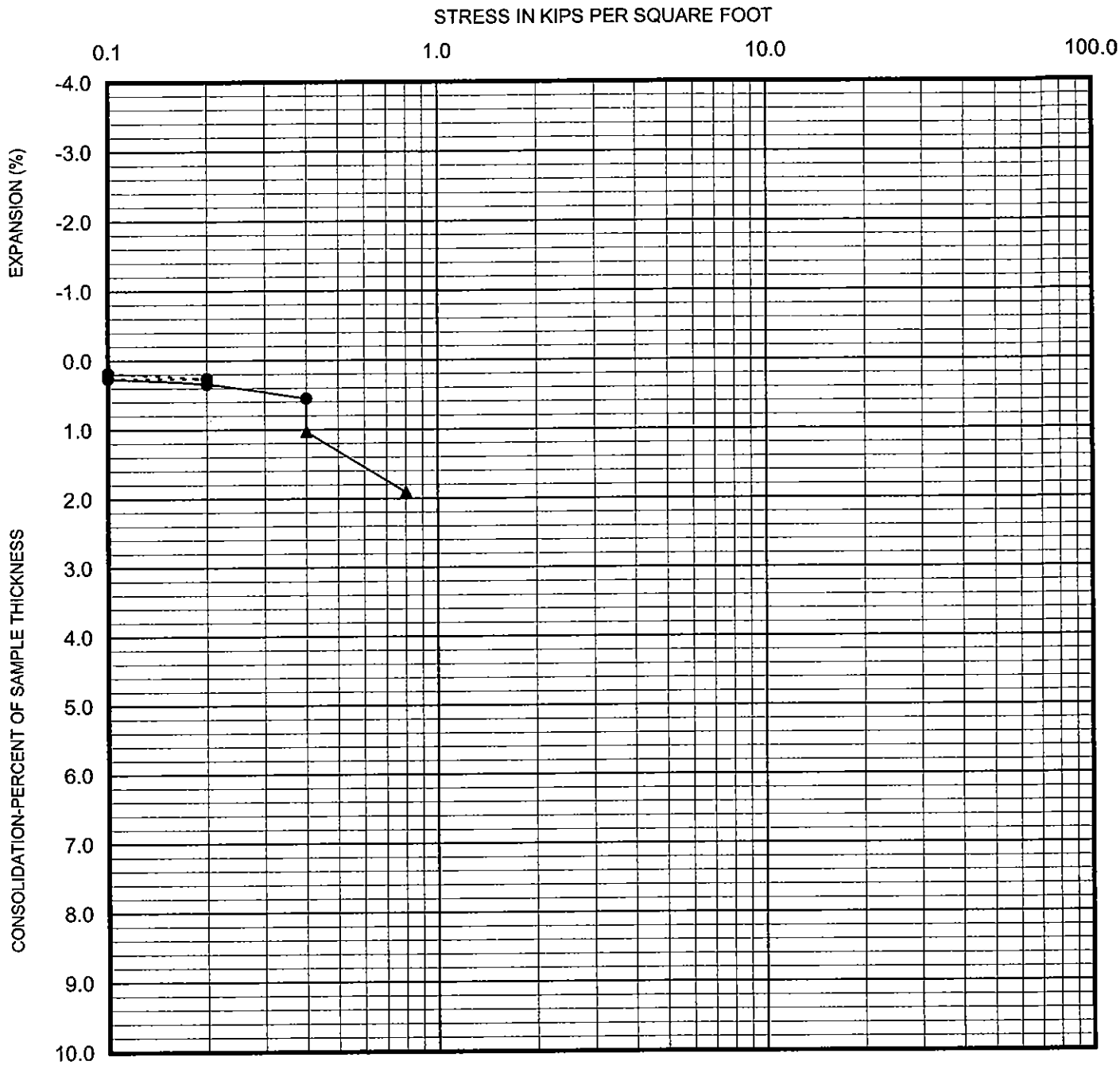
CONSOLIDATION TEST RESULTS

SUPERIOR FIRE STATION
GOLF COURSE ROAD & MARY DRIVE
SUPERIOR, ARIZONA

PROJECT NO.	DATE	FIGURE
600777001	3/04	B-5

CNB1@6-7.4



Seating Cycle	Boring No. B-2
Loading Prior to Inundation	Depth (ft.) 3.5-4.8
Loading After Inundation	Soil Type SC
Rebound Cycle	PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435-96

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CONSOLIDATION TEST RESULTS

SUPERIOR FIRE STATION
GOLF COURSE ROAD & MARY DRIVE
SUPERIOR, ARIZONA

PROJECT NO.	DATE	FIGURE
600777001	3/04	B-6

CNB2@3.5-4.8

EXPANSION INDEX TEST RESULTS

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	EXPANSION POTENTIAL
B-1	0-5	9.9	107.8	21.2	0.038	38	Low
B-2	0-5	11.2	110.4	18.3	0.0134	13	Very Low

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4829-95

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EXPANSION INDEX TEST RESULTS

SUPERIOR FIRE STATION
GOLF COURSE ROAD & MARY DRIVE
SUPERIOR, ARIZONA

PROJECT NO.	DATE	FIGURE
600777001	3/04	B-7

CORROSIVITY TEST RESULTS

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH *	RESISTIVITY * (ohm-cm)	WATER-SOLUBLE SULFATE CONTENT IN SOIL ** (%)	CHLORIDE CONTENT *** (ppm)
B-2	0-5	7.7	510	0.01	690

* PERFORMED IN GENERAL ACCORDANCE WITH ADOT TEST METHOD ARIZ 236b
** PERFORMED IN GENERAL ACCORDANCE WITH ADOT TEST METHOD ARIZ 733
*** PERFORMED IN GENERAL ACCORDANCE WITH ADOT TEST METHOD ARIZ 722

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CORROSIVITY TEST RESULTS

SUPERIOR FIRE STATION
GOLF COURSE ROAD & MARY DRIVE
SUPERIOR, ARIZONA

PROJECT NO.	DATE	FIGURE
600777001	3/04	B-8